IMPROVING THE MOMENT CAPACITIES OF HOLLOW FLANGE COLD-FORMED LITESTEEL BEAMS USING WEB STIFENERS

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Abstract

The LiteSteel Beam (LSB) is a new hollow flange section developed in Australia with a unique geometry consisting of torsionally rigid rectangular hollow flanges and a relatively slender web. The LSB is subjected to a relatively new Lateral Distortional Buckling (LDB) mode when used as flexural members. Unlike the commonly observed lateral torsional buckling, lateral distortional buckling of LSBs is characterised by cross sectional change due to web distortion. Lateral distortional buckling causes significant moment capacity reduction for LSBs with intermediate spans. Therefore a detailed investigation was undertaken to determine the methods of reducing the effects of lateral distortional buckling in LSB flexural members. For this purpose the use of web stiffeners was investigated using finite element analyses of LSBs with different web stiffener spacing and sizes. It was found that the use of 5 mm steel plate stiffeners welded or screwed to the inner faces of the top and bottom flanges at third span points considerably reduced the lateral distortional buckling effects in LSBs. Suitable design rules were then developed to calculate the enhanced elastic lateral distortional buckling moments and the higher ultimate moment capacities of LSBs with the chosen web stiffener arrangement. This paper presents the details of this investigation and the results.

1. INTRODUCTION

Cold-formed steel members are widely used in building applications due to their light weight nature and high strength characteristics. Recently an advanced cold-formed section, called the LiteSteel beam (LSB), was introduced in Australia as an alternative and improved section to replace the conventional cold-formed C- and Z- sections and smaller hot-rolled I- and channel sections (Dempsey, 1990). The LSB section is made of two torsionally rigid closed flanges and a slender web (Figure 1). The high strength steel material used for LSBs is DuoSteel grade with a web yield stress of 380 MPa and a flange yield stress of 450 MPa. There are 13 LSB sections with their depth in the range of 125 to 300 mm while the flange width varies from 45 mm to 75 mm. Their thickness varies from 1.6 to 3 mm. The LiteSteel Beams (LSBs) with intermediate and long spans are subjected to lateral distortional and lateral torsional buckling, respectively. Lateral distortional buckling occurs due to the presence of torsionally rigid rectangular flanges and a slender web. Simultaneous lateral displacement, section twist and web distortion occur during this lateral distortional buckling of LSB as seen in Figure 1, which significantly reduces the moment capacity of LSBs with intermediate spans (Anapayan et al., 2010). Such moment capacity reduction can be avoided if the observed web distortion in LSBs is eliminated or reduced.

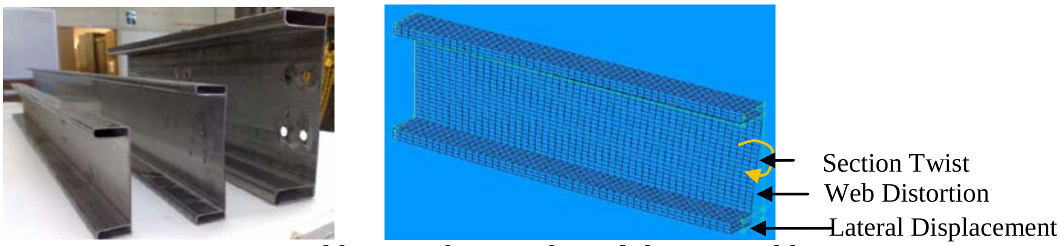


Figure 1: LiteSteel beam subject to lateral distortional buckling

(a) Support without FTR (b) Local flange twist (c) Support with FTR

Figure 2: Twist Restraint at the Supports

Avery and Mahendran (1997) have shown that the use of web stiffeners reduces web distortion and hence improves the moment capacity of a doubly symmetric hollow flange section. They found that simple plate stiffeners welded or screw fastened to only the top and bottom flanges were able to reduce the web distortion and thus improve the lateral buckling moment capacities. However, Kurniawan's (2005) investigations on LSBs produced conflicting outcomes. His experimental studies based on quarter point loading showed that the use of web stiffeners did not significantly improve the moment capacity of LSBs although his finite element analyses gave improved buckling moment capacities. This may be due to the lack of flange twist restraints at the supports in his lateral buckling tests (Figure 2(a)). It appears that the use of web side plates alone was unable to provide the required flange twist restraint (FTR) as seen by the local flange twists at the supports in Figure 2(b). This does not produce the ideal pinned conditions with full twist restraint to the entire section. Transverse web stiffeners were provided at the supports in Anapayan et al.'s (2010) lateral buckling tests to provide FTR at the supports (Figure 2(c)). The difference between the support conditions in Kurniawan's (2005) experimental and finite element analyses might have caused the moment capacity differences. Therefore a thorough investigation was undertaken to investigate the use of web stiffeners to improve the lateral distortional buckling and ultimate strength of LSBs.

For this purpose Anapayan and Mahendran's (2012) validated finite element models of LSBs were used by including the required web stiffeners at the supports and appropriate locations within the span. It is important to investigate the reasons for the conflicting outcomes of Kurniawan (2005) and then to determine the most suitable and cost-effective type, size and spacing of the required web stiffeners that will provide improved lateral distortional buckling capacities for LSBs. This was first undertaken using a series of elastic buckling analyses. Both elastic buckling and non-linear analyses were then undertaken for the chosen web stiffener arrangement and suitable design rules were developed. This paper presents the details of this investigation and the results.

2. ELASTIC BUCKLING ANALYSES

2.1 Finite element models

LSBs stiffened with plate web stiffeners were considered in this research. Two types of finite element models were used, namely, ideal and experimental finite element models as shown in Figure 3. Ideal models of LSBs were based on ideal simply supported conditions and a uniform moment. Ideal simply supported boundary conditions were implemented by fixing the vertical and lateral deflections and twist of the section at the supports. Experimental finite element models were used to simulate the LSBs as used in the lateral buckling tests with quarter point loading. Anapayan and Mahendran (2012) provides the details of these FE models of LSBs without web stiffeners that have been validated using lateral buckling test results. These validated models were modified by including the required web stiffeners in this research. Steel plates with 5 mm thickness and a yield stress of 300 MPa were considered at the supports and one third points of the beam span. The plate stiffeners at the supports provided the required flange twist restraint in the experimental finite element model. Figure 4 shows the experimental finite element model of LSB with plate web stiffeners.

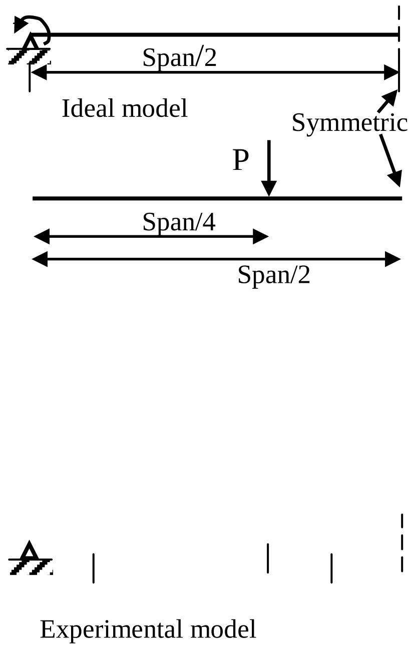


Figure 3: Finite element models

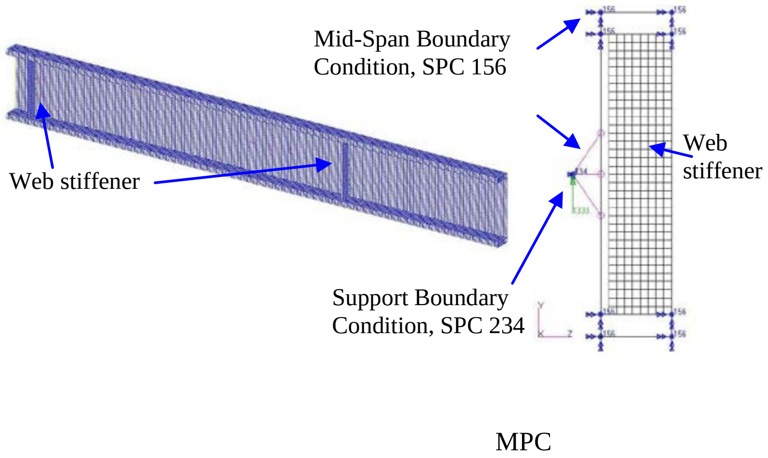


Figure 4: Experimental FE model of LSB

Shell elements of 5 mm width and 10 mm length were used as shown in Figure 4. It shows the various plates used in the experimental FE model with web stiffeners at supports and suitable locations within the span (third points), and also the web side plates used at the supports. The cross sectional view of LSB with web stiffeners, which includes the support and mid-span boundary conditions and loading, is also shown in Figure 4. The web stiffeners were connected to the inner flange surface by a process of “equivalencing” the nodes of the web stiffener plate and the nodes of the inner surface of the flange so that the web stiffener plate and the flange can act as an integral member. The welding process was not modelled as it was decided to recommend a “tack” weld. A 5 mm gap was provided between the stiffener and the web element due to the corners present in LSBs. Elastic buckling analyses were undertaken with varying arrangements of web stiffeners to investigate the need for web stiffeners at the supports, ie. no web stiffeners, web stiffeners at the supports providing FTR, web stiffeners at third points within the span, and web stiffeners at the supports and third span points.

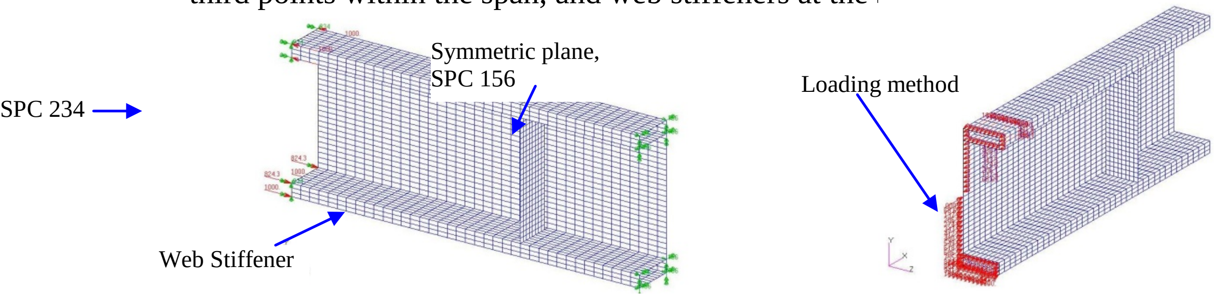


Figure 5: Ideal FE model of LSB

Figure 5 shows the typical ideal finite element model of LSBs with web stiffeners. Flange twist restraints at the supports were not modelled explicitly using web stiffeners. Instead they were included in the models via idealised simply supported boundary conditions which provide full twist restraint to the entire section at the supports. The section twist was restrained by fixing the X axis rotation (SPC 4) while the vertical and lateral displacements were also fixed at all the nodes of the end supports of LSB.

To simulate a uniform end moment on the ideal FE model, linear forces were applied at every node of the beam end, where the nodes above the middle of the web were subject to tensile forces while the nodes below the middle of the web were subject to compressive forces. The force at the middle of the web was zero and was linearly increased within the cross section as shown in Figure 5. A tensile force of 1000N and a compressive force of 1000N were applied at the nodes on the top and bottom faces of LSB cross section.

2.2 Elastic buckling analysis results

Table 1 presents the elastic lateral distortional buckling moments of LSBs with varying web stiffener arrangements from both experimental and ideal finite element models. It includes the elastic lateral distortional buckling moments (M_{od}) from the ideal FE model, the elastic torsional buckling moment M_o, and the values of M_{od} with various arrangements of web stiffeners as obtained from the experimental FE models. Elastic lateral torsional buckling moment equation was used to calculate M_o shown in Table 1.

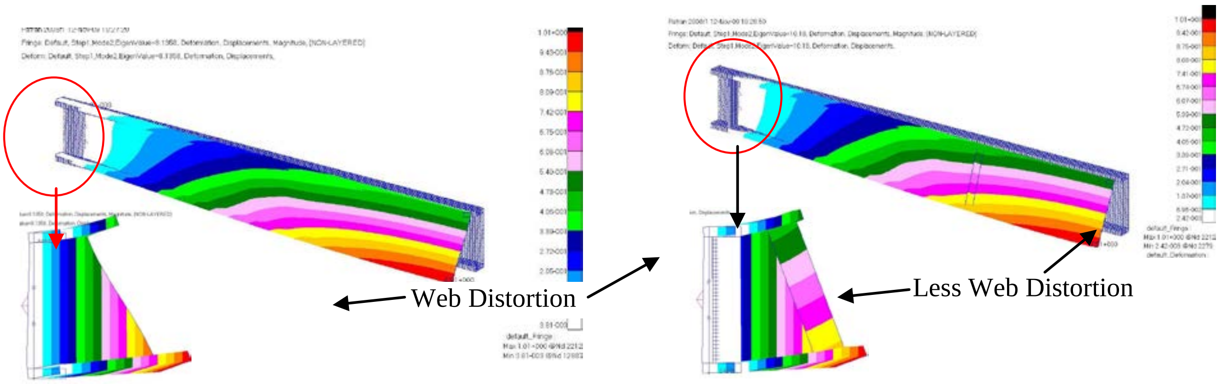
Table 1: Elastic lateral distortional buckling moments of LSBs with web stiffeners

LSB Sections	Span (mm)	M_o (kNm)	M_{od} (kNm) Ideal Model	M_{od} (kNm) Experimental FE Model			
				No WS (a)	WS_s (b)	WS_{TP} (c)	WS (d)

300x60x2.0 LSB	3000	33.95	22.99	22.05	23.80	25.02	29.06
	4000	24.66	18.36	17.55	19.46	19.66	22.81
200x45x1.6 LSB	3000	10.68	8.33	8.14	8.95	9.00	10.18
	4000	7.89	6.67	6.43	7.05	6.89	7.63
150x45x2.0 LSB	2000	18.35	14.52	12.15	14.30	13.01	15.81
	3000	11.96	10.48	9.01	10.42	9.39	10.93

WS_s – Web stiffeners at the supports providing flange twist restraint,
WS_{TP} – Web stiffeners at third points within the span,
WS – Web stiffeners at the supports and third span points.

As seen in Table 1, elastic lateral distortional buckling moments (M_{od}) were increased by 19 to 32% for the LSBs and spans considered here when web stiffeners were used at third points within the span and supports (WS). However, when the web stiffeners were used only at third span points without any stiffeners at the supports (without flange twist restraint at the support - WS_{TP}), the improvement to elastic lateral distortional buckling moment was not significant. This difference is small in some cases (200x45x1.6 LSB with 4 m span) since these LSBs exhibit lateral torsional buckling with very small web distortion. Table 1 results thus demonstrate the need to use web stiffeners at both the supports and third span points.



(a) No web stiffeners (b) Web stiffeners at the supports and third span points

Figure 6: Lateral distortional buckling modes of LSBs with various stiffener arrangements

Figures 6 (a) and (b) show the elastic lateral distortional buckling modes obtained for 200x45x1.6 LSBs. They clearly demonstrate that web stiffeners at the supports significantly reduced the local flange twist at the support. Comparison of figures shows that web distortion was reduced when web stiffeners were used at third span points. Although the web stiffeners at third span points reduced the web distortion, the use of web stiffeners at the supports is also important as this further improved the moment capacities by avoiding local flange twist.

Based on these elastic buckling analysis results, it is concluded that the use of web stiffeners at every third point within the span can effectively improve the lateral distortional buckling moment capacity of LSBs provided web stiffeners are also used at the supports. This simulates the idealised simply supported conditions with full twist restraint. These results also provide the explanation why experimental and numerical analyses gave conflicting outcomes in relation to the buckling capacity improvements due to web stiffeners in Kurniawan (2005).

Having confirmed the effectiveness of using web stiffeners welded to the inner faces of top and bottom flanges in improving the lateral distortional buckling capacities of LSBs, optimum size and spacing of the required plate web stiffeners was investigated as described next.

2.3 Optimum web stiffener configuration

For this purpose, a series of elastic buckling analyses was conducted using the ideal FE model to investigate the lateral distortional buckling moment capacities of LSBs as a function of web stiffener plate thickness (5 mm) and spacings of span/2, span/3 and span/4. Three different LSBs with three intermediate spans (2, 4 and 6m) were considered. Elastic lateral distortional buckling moments (M_{od}) increased with decreased stiffener spacing. The ratio of M_{od} values for span/3 and span/2 was about 1.05 for intermediate spans while they were about 1.02 for span/4 and span/3. This indicates that the degree of improvement to M_{od} is not significant when the web stiffener spacing was reduced from span/3 to span/4. An additional web stiffener thus only provides about 2% increase in M_{od}. Therefore span/3 was considered to be the optimum web stiffener spacing based on both member capacity and cost.

For this spacing of span/3, the effects of web stiffener thickness were then investigated. Four thicknesses of 3 mm, 4 mm, 5 mm and 10 mm were considered for seven LSBs with varying spans of 2, 4, 6, 8 and 10 m. The buckling moment capacities increased with increasing web stiffener thickness for intermediate spans while this increment was very small for long spans for which web distortion is small. Buckling moment capacity improvement was only about 1.4% for intermediate spans when 5 mm web stiffeners were replaced with 10 mm web stiffeners. Therefore 10 mm web stiffeners cannot be justified. Elastic lateral distortional buckling capacity improvement was less than 1% when web stiffener thickness was increased from 3 to 5 mm. The corresponding ultimate moment capacity increase will also be less than 1%. Therefore it was concluded that web stiffeners with a thickness in the range of 3 to 5 mm can be used without much difference in buckling and ultimate moment capacities of LSBs.

Based on these results, it was decided to use 5 mm thick plate web stiffeners, welded to the inner faces of top and bottom flanges at third points within the span and supports as the optimum web stiffener configuration. However, thinner web stiffeners (3 or 4 mm) could be considered for smaller LSBs such as 125x45x1.6/2.0 LSB and 150x45x1.6/2.0 LSB.

Mahendran and Avery (1997) showed that screw-fixed or welded web stiffeners are equally effective in improving the lateral distortional buckling capacities of hollow flange sections. Kurniawan's (2005) experiments of LSBs with screw-fixed web stiffeners (Figure 8) also confirmed this. Hence it is concluded that web stiffeners can also be screw-fixed as shown in Figure 8 to the inner faces of top and bottom flanges of LSBs. It was then decided to obtain the elastic lateral distortional buckling and nonlinear ultimate moments of all the 13 LSBs stiffened with 5 mm plate web stiffeners using the ideal FE model shown in Figure 5.

Table 2: Comparison of elastic lateral buckling moments of stiffened LSBs

LSB Sections	Span (mm)	M_o (kNm)	M_{od} (kNm)	M_{odw} (kNm)	M_o/M_{od}	M_{odw}/M_{od}	M_{odw}/M_o
300x75x3.0 LSB	2000	150.3	97.87	132.28	1.54	1.35	0.88
	4000	67.35	52.37	61.24	1.29	1.17	0.91
	6000	43.87	38.0	40.69	1.15	1.07	0.93
	8000	32.63	29.71	30.7	1.10	1.03	0.94
	10000	26	24.29	24.72	1.07	1.02	0.95

2.4 Equation to predict elastic lateral distortional buckling moment of stiffened LSBs

Table 2 presents and compares the elastic lateral torsional buckling moments (M_o), elastic lateral distortional buckling moments without web stiffeners (M_{od}) and the elastic lateral distortional buckling moments with web stiffeners (M_{odw}) for one LSB section. As expected the use of web stiffeners improved the elastic lateral distortional buckling moment capacities for all the LSB sections as reflected by the high M_{odw}/M_{od} ratios and the M_{odw}/M_o ratios approaching 1.0 (>0.88 in Table 2). In most cases of LSBs with intermediate spans, this ratio was in the range of 0.90 to 0.97. A relationship between M_o and M_{odw} was developed in order to calculate M_{odw} without undertaking any finite element analyses. For this purpose, the ratio of M_{odw}/M_o was plotted against non-dimensional member slenderness λ in Figure 7. The first yield moments (M_y) of LSBs were calculated using a yield stress of 450 MPa.

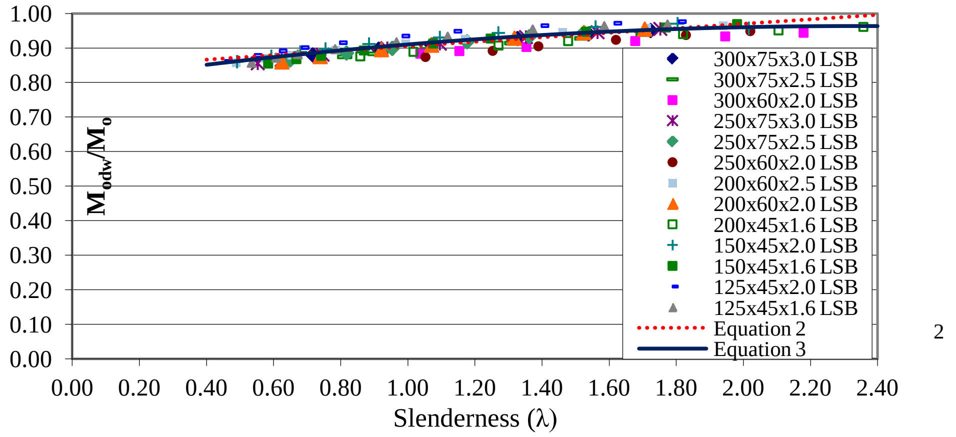


Figure 7: M_{odw}/M_o versus Slenderness for LSBs with Web Stiffeners

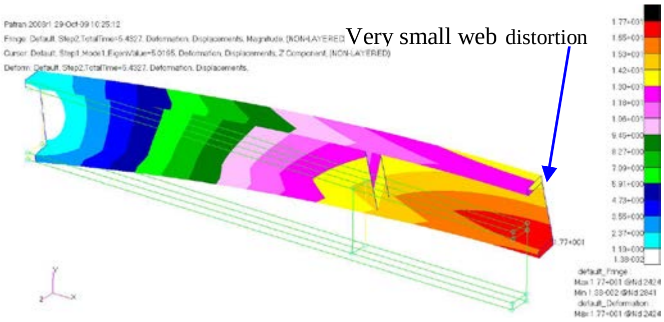
Two equations were developed. Equation 2 is a linear equation while Equation 3 is a second order polynomial equation and the relevant curves are shown in Figure 7.

$$\lambda = (M_y/M_o)^{1/2} \tag{1}$$
$$M_{odw}/M_o = 0.065\,\lambda + 0.84 \tag{2}$$
$$M_{odw}/M_o = -\,0.30\,\lambda^2 + 0.14\,\lambda + 0.80 \tag{3}$$

The ratios of FEA to predicted buckling moment ratios were obtained and the mean and COV values were calculated for both equations. Equation 2 has a mean FEA to predicted value of 1.00 and a COV of 0.017 while those for Equation 3 are 1.00 and 0.015. This indicates that both equations are accurate to predict the elastic lateral distortional buckling moments of LSBs with the chosen optimum web stiffener configuration.

3. ULTIMATE MOMENT CAPACITIES OF STIFFENED LSBS

In order to predict the ultimate moment capacities of stiffened LSBs subjected to lateral buckling, non-linear finite element analyses were conducted for the available 13 LSBs. Ideal finite element models used in the elastic lateral distortional buckling analyses of LSBs with the optimum web stiffener configuration of 5 mm stiffeners at third span points was adopted. Nominal LSB sizes and yield stresses were used in the models with relevant initial geometrical imperfection (L/1000) and residual stresses. The imperfection values, direction and the residual stress distribution are the same as used by Anapayan and Mahendran (2012) in their investigation of unstiffened LSBs. The lateral buckling failure mode of a 2 m span 150x45x2.0 LSB obtained from the non-linear finite element analysis is shown in Figure 8. It also shows that web distortion has been significantly reduced by the use of web stiffeners. The ultimate moment capacities of selected LSBs and spans from FEA with and without web stiffeners are presented in Table 3. The ultimate moment capacities were found to be about the same even when smaller stiffener thicknesses of 3 mm and 4 mm were used (<1%). Therefore the results are equally applicable to stiffener thicknesses in the range of 3 to 5 mm.



Lateral buckling failure mode

Figure 8: Lateral buckling failure mode of a stiffened 150x45x2.0 LSB

Table 3: Ultimate moment capacities of LSBs

LSB Sections	Span (mm)	M_u (kNm)		% increase
		without WS	with WS	
300x60x2.0 LSB	3000	17.77	22.62	27.32
	4000	14.98	17.98	20.00
	6000	11.57	12.81	10.71
	8000	9.43	10.02	6.24
200x45x1.6 LSB	1500	9.85	12.37	25.55
	2000	8.43	10.49	24.56
	3000	6.85	7.88	15.11
	4000	5.76	6.26	8.55
	6000	4.33	4.51	4.21

As seen in Table 3, the increase in the ultimate moment capacity of LSBs is high for intermediate spans (up to 27%) while it is small for long spans. This is as expected since lateral torsional buckling is the dominant buckling mode for long spans. In summary the ultimate moment capacity results on Table 3 demonstrate the significant member moment capacity improvements for LSBs when web stiffeners are used as recommended here.

4. DESIGN RULES TO PREDICT THE ULTIMATE MOMENT CAPACITY

The ultimate moments from FEA were non-dimensionalised and compared with the design curve developed by Anapayan and Mahendran (2012) for LSBs without web stiffeners.

For $\lambda_d \leq 0.54$: $M_c = M_y$ (4a)

For $0.54 < \lambda_d < 1.74$: $M_c = M_y\,(0.28\,\lambda_d^2 - 1.20\,\lambda_d + 1.57)$ (4b)

For $\lambda_d \geq 1.74$: $M_c = M_y\left(\dfrac{1}{\lambda_d^2}\right)$ (4c)

where, $\lambda_d = \sqrt{\dfrac{M_y}{M_{od}}}$ and $\lambda_{dw} = \sqrt{\dfrac{M_y}{M_{odw}}}$ (4d)

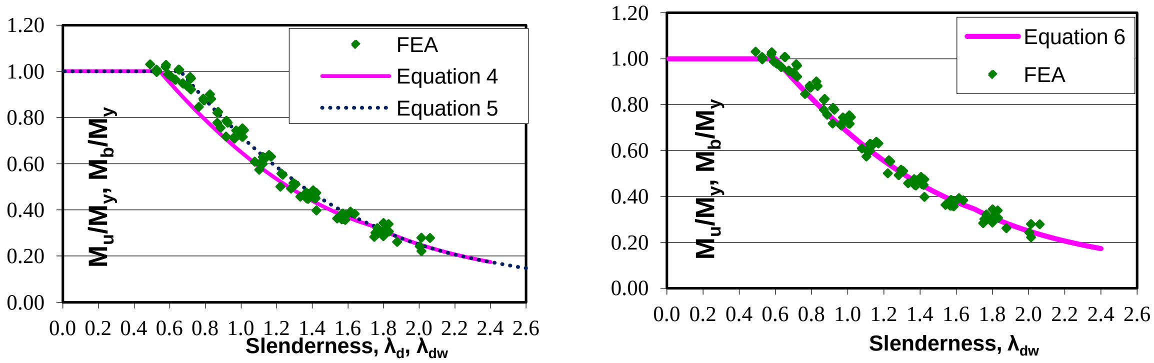


Figure 9: Comparison of ultimate moment capacities with proposed equations

Figure 9 shows the comparison of FEA results with Equation 4 modified by replacing λ_d with λ_{dw} to include the effect of plate web stiffeners. It shows that Equation 4 is conservative in predicting the ultimate moments of LSBs with web stiffeners as most of the FEA data points are above the design curve. The ratios of the ultimate moments from FEA and Eq.4 were calculated and the mean of this ratio was 1.06 and the COV was 0.053. The capacity reduction factor Φ was calculated using the recommended AISI procedure (AISI, 2007). It was 0.96 compared with the recommended capacity reduction factor of 0.90 for cold-formed steel members in bending. Hence it is concluded that Equation 4 is conservative.

Next the use of equations developed for the back to back LSBs were considered. The design equations for the ultimate moment capacities of back to back LSBs without web stiffeners were developed by Jeyaragan and Mahendran (2009) and are given next.

For $\lambda_d \leq 0.65$: $M_c = M_y$ (5a)

For $0.65 < \lambda_d < 1.80$: $M_c = M_y\,(0.28\,\lambda_d^2 - 1.29\,\lambda_d + 1.73)$ (5b)

For $\lambda_d \geq 1.80$: $M_c = M_y\left(\dfrac{1}{\lambda_d^2}\right)$ (5c)

Figure 9 shows that most of the FEA data points are below the design curve based on Equation 5. The mean of the ratio of ultimate moment capacities from FEA and this equation was found to be 0.97 and the associated COV was 0.049 with a Φ factor of 0.88. This indicates that Equation 5 is also not suitable to predict the ultimate moment capacities of stiffened LSBs subject to lateral buckling. Therefore a new design equation (Eq.6) was developed using the available FEA ultimate moments (92 results).

For $\lambda_{dw} \leq 0.60$: $M_c = M_y$ (6a)

For $0.60 < \lambda_{dw} < 1.70$: $M_c = M_y\,(0.29\,\lambda_{dw}^2 - 1.26\,\lambda_{dw} + 1.65)$ (6b)

For $\lambda_{dw} \geq 1.70$: $M_c = M_y\left(\frac{1}{\lambda_{dw}^2}\right)$ (6c)

The mean and COV of the ratio of ultimate moment capacities from FEA and Equation 6 were calculated to be 1.02 and 0.050, respectively. The Φ factor was found to be 0.92, which is slightly higher than the recommended value of 0.90, and is acceptable.

5. CONCLUSIONS

This paper has presented the details of an investigation on the effects of using plate web stiffeners on the lateral distortional buckling moment behaviour and capacity of LSBs. Various types of web stiffener configurations including their size and spacing were considered using a series of elastic buckling analyses. It was found that 5 mm thick steel plates welded or screw fixed to the inner surfaces of the top and bottom flanges at the beam supports and at third points within the span considerably improved the lateral distortional buckling moment capacities of LSBs. The use of web stiffeners reduced the level of web distortion considerably and thus allowed the LSB flexural members to achieve higher moment capacities. Thinner web stiffeners (3 or 4 mm) can also be considered to be equally effective for smaller LSBs. Suitable equations were developed to calculate the elastic lateral distortional buckling moments of stiffened LSBs. The ultimate moment capacities of stiffened LSBs were compared with the developed design rules for single and back to back LSBs without web stiffeners and a new design rule was developed to accurately predict the ultimate moment capacities of LSBs with web stiffeners subject to lateral buckling.

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